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Autor: Vandegans, Didier

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Use of threaded studs in joints between I-beam and R.H.S. column

Didier VANDEGANS
Civil Engineer
CRIF Steel Construction Dpt.
Liège
Belgium

Didier Vandegans (1971) got his Civil Engineering Degree at the University of Liège in 1994. His main domain of activity is the semi-rigid connections.

Summary

This paper shows that the component method, described in Annex J of Eurocode 3 can be applied in the case of studded joints, by using different existing models. Numerical simulations make it possible to determine the deformation of the bended face of the section, on which the studs are fixed. Therefore the curves of behaviour of these joints can be drawn and compared with the experimental curves.

1. Introduction

The concrete-filled R.H.S. (Rectangular Hollow Section) technique has many advantages in the building domain. The bearing load, the stiffness of joints between beams and columns and the fire stability are increased, the floor space required is smaller, the aesthetic is improved, the maintenance is easier, and in comparison with concrete column, no shuttering is needed.

The stiffness of joints with steel beams or composite steel-concrete beams, and their resistance to bending moment are relatively high. In order to calculate the resistance of different joint elements, some codes or models are developed. However, no model exists to determine the deformability of the face of the hollow section when it is in bending.

The aim of this paper is to show that, with the rules given in Annex J of Eurocode 3 (EC3) [1], based on the component method, with different design models and with numerical simulations, it is possible to determine the characteristics of the behaviour curve of joints between a I-beam and a concrete filled R.H.S. column.
Furthermore, experimental tests show that important membranar effects develop within the face of the hollow section. Preliminary studies are now carried out to see if this effect can be neglected with accuracy.

2. The stud technique.

The presence of concrete within the hollow section make it impossible to fix a connection element with bolts on the face of this section. The stud technique is useful to solve this problem. This technique consists in welding with the help of a special gun, a threaded stud on the face of the section on which the connection is to be realised [3]. The other elements of the connection are fixed to the studs with nuts, as done for classical bolts. The studs are then subjected to traction and shearing, as for usual joints. The studs to be used are threaded studs with reduced base. So, the welds have approximately the same diameter as the threaded part.

3. Test results.

Eight tests have been performed in the laboratory. Four different joint configurations were considered (web cleats, extended and flush end-plates, flange cleats). The full description of the experimental program is given in [5].
The experimental curves, as well as the rigid and pinned classification domain, are given in figure 1. For the classification, a braced frame is considered with a beam span equal to 6 meters.

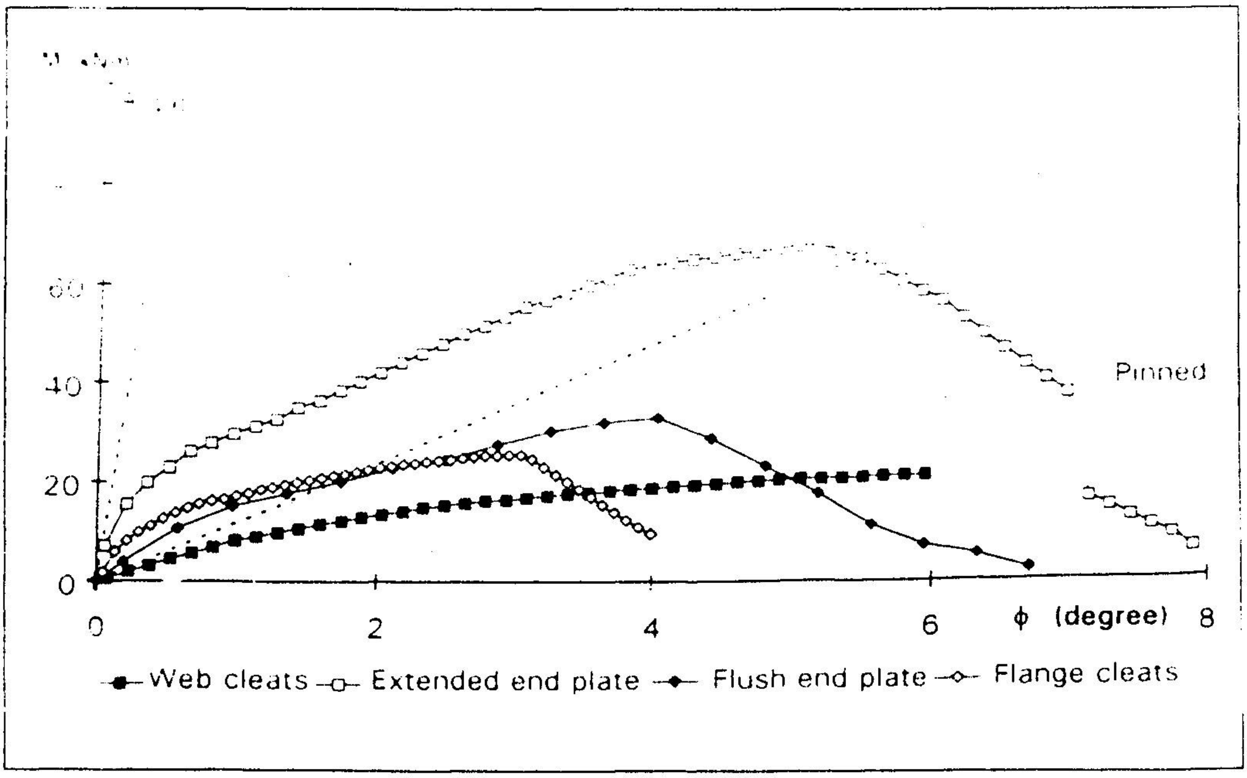

Fig. 1. Experimental curves and classification

4. Method of calculation used.

4.1. Annex J of EC3 method.

For the design of the joint, the approach described in Annex J of Eurocode 3 is used. This is based on the component method, which considers a joint not as a unit, but as a set of individual components each with its own strength and stiffness. EC 3 Annex J allows to calculate the characteristics of the following components :

- end-plate in bending;
- flange cleats in bending;
- flange of the beam in compression;
- web of the beam in tension;
- flange of the beam in bearing;
- bolts in shearing;
- studs in tension.

4.2 Naveau model.

An earlier research carried out by NAVEAU in CRIF in Belgium [4] has established some design rules for the resistance of the face of the rectangular hollow section. These rules cover the following modes of failure :

- shearing of the face of the section :

$$N_{max} \leq 0{,}95.\pi.d.t.\frac{f_{yt}}{\sqrt{3}}/\gamma_{mo} \qquad (1)$$

- lamelar pull out :

$$N_{max} \leq 0{,}95.\frac{\pi\, d^2}{4}\, f_{yt}/\gamma_{mo} \qquad (2)$$

f_{yt} is the yield strength of the hollow section, t is the thickness of the face, d is the nominal diameter of the studs.

4.3 Gomes model.

In order to calculate the resistance of the R.H.S bended face, the so-called "Gomes Model" is used. Gomes has studied the weak axes beam to column joints and has deduced a model to design the column web. A part of this model predicts a local failure of the column web due to the bolts in tension. He substitutes the plastic mechanism of the web by an equivalent rectangle of b x c dimensions as shown on figure 2.

The local resistance of the web is :

$$F_{local} = M_{pl}.\alpha.k \qquad (3)$$

in which M_{pl} is the plastic moment of the web.

$$M_{pl} = \frac{1}{4} t_w^2 f_{yw} \tag{4}$$

$$\alpha = \frac{4}{1-b/L}\left(\pi\sqrt{1-b/L} + 2c/L\right) \tag{5}$$

$$k = \begin{cases} 1 & \text{if } (b+c)/L \geq 0.5 \\ 0.7 + 0.6\,(b+c)/L & \text{if } (b+c)/L \leq 0.5 \end{cases} \tag{6}$$

The validity range of this model is as follows :

$$\begin{cases} b/L < 0.8 \\ 0.7 \leq h/(L-b) \leq 10 \end{cases} \tag{7}$$

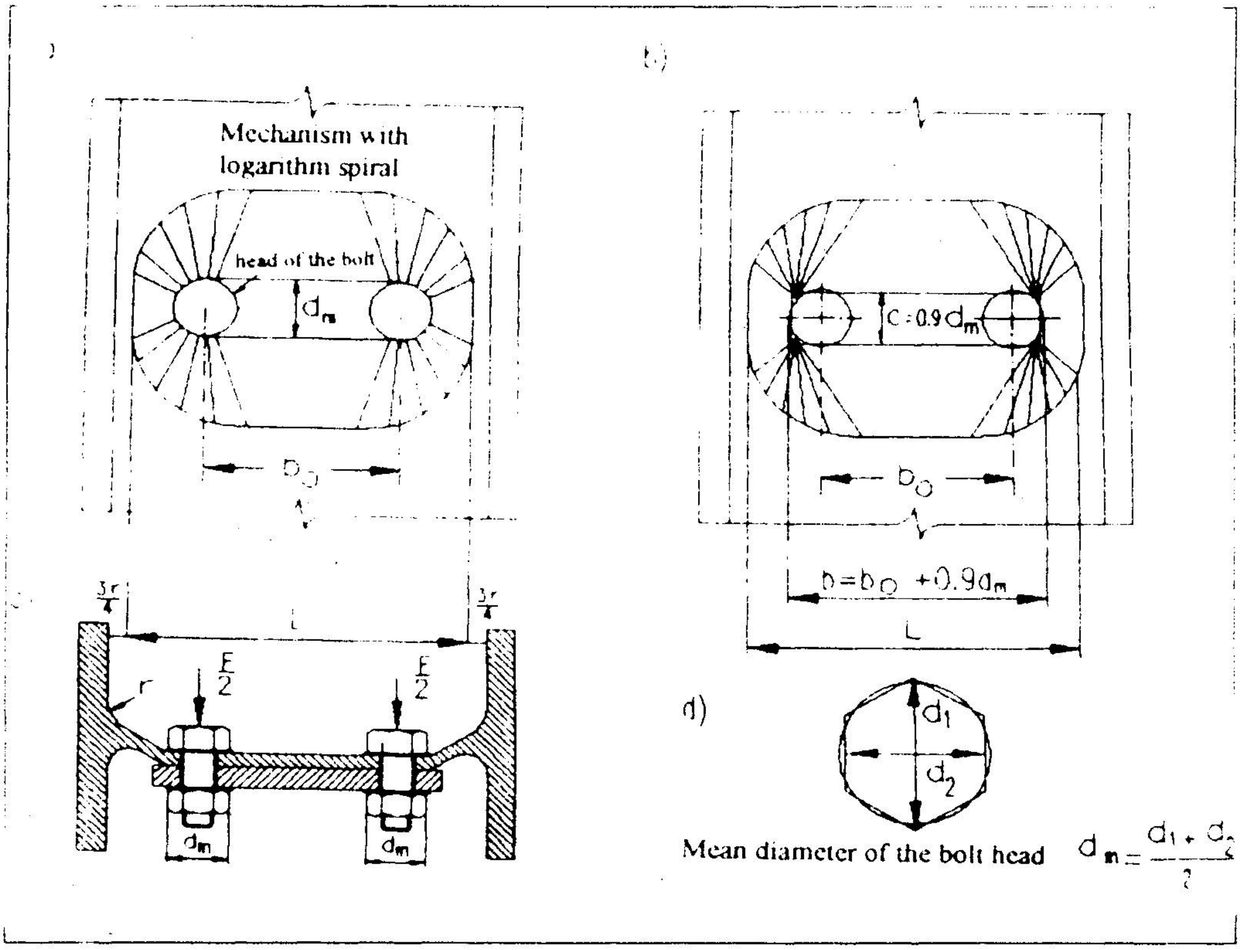

Fig. 2. Local mechanism for bolted connections:
i) yield line pattern
ii) yield line pattern for equivalent rectangle b x c
iii) section view
iv) mean diameter of the bolt head (or nut)

This model can be extended to the bended face of the rectangular hollow section. If two stud rows are present, the model might also be extended, by taking group plastic mechanism into account.

4.4 Numerical simulations.

So far, no model exists to evaluate the deformability of the bended face of the hollow section. Numerical simulations, with shell elements, have been done with a non-linear finite elements program (FINELG [9]), in order to find a numerical value of the deformation.

5. Hypotheses.

During the calculation, the following elements have been considered:

1. The presence of concrete stiffens the sheared part of the connection very strongly and increases the compression zone resistance of the joint considerably. Because of lack of information, it is not possible to calculate with a high degree of precision the shear and compression resistance and the corresponding deformations. But it is obvious that the resistances which would have been found are largely higher than those of the other components, and in addition, the deformability of these zones is extremely low.

2. Lateral faces of the section are submitted to tensile forces, acting on a defined length corresponding to the efforts diffusion length. The value of this diffusion length cannot be determined with a high degree of precision. However, the "web" of the R.H.S. may be considered as largely over-dimensioned in comparison with its equivalent in a I-beam. The reason is that, in this case, the faces of the section have the same thickness, as opposed to the difference in thickness between the flanges and the web of an I-beam. In addition, adherence may occur between the lateral faces and the concrete. Therefore, this source of deformation is supposed to be negligible. This hypothese is confirmed by the numerical simulations.

3. When pulled out by the studs, the face of the section comes out of its plane. The angles of the rectangular section are rigid, and thus, while the section is empty, the deformation of one induces a deformation of the other lateral faces, as shown on the figure 3. The presence of concrete within the hollow section prevents these lateral faces from deforming and therefore reduces the deformation of the face connected to the studs when they are submitted to traction. This fact stiffens this component, and consequently, during the numerical simulations, the face of the section is modelised by means of a infinitely long plate, imbedded on 2 faces and subjected to two "concentrated" forces. The reason why a numerical simulation has to be done is that, to our knowledge, no analytical solutions are available.

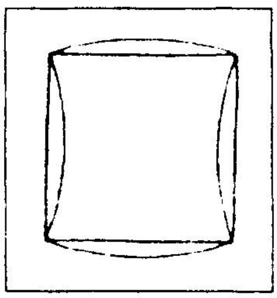

Fig. 3. Free out of plane deformation of the section's faces

6. Results of the calculation.

Without giving all the details, which can be found in [5], the results of the stiffness and strength calculations are given in table 1, as well as the corresponding mode of failure.

	M_{Rd} (in kN.m)	M_u^{exp} (in kN.m)	Failure mode	S_{ji}^{calc} (in kN.m/deg)
Extended end-plate	35.2	67.10	Face of the section	78.97
Flush end-plate	16.5	33.15	"	34.85
Flange cleats	17.7	25.50	Cleat	37.23

Table n° 1 : Stiffness and strength calculation results

Figure 4 shows the experimental curves and their equivalent given by calculation.
The experiments show that important membranar effects develop in the bended face of the column. Because of the importance of these effects, the ratio between the initial and secant stiffnesses, S_{ji} and S_{js}, as well as between the elastic moment M_e and the design moment resistance M_{Rd}, have to be slightly modified.

The formulae given in EC 3 Annex J are as follows :

$$S_j = \frac{E.z^2}{\mu.\sum_i \frac{1}{k_i}} \qquad (8)$$

where E = the Young modulus;
z = the lever arm of the external forces;
k_i = the stiffness of the individual components;
μ = the stiffness ratio.

$$\mu = \left[1.5\,\frac{M_{j.Sd}}{M_{j.Rd}}\right]^{\psi} \qquad (9)$$

and ψ = 2.7 for welded and bolted end-plate joints;
= 3.1 for flange cleated joints.

For the determination of the secant stiffness S_{js}, $M_{j.Sd}$ equals $M_{j.Rd}$ and thus

$$\mu = (1.5)^{\psi} \qquad (10)$$

$$\text{and } S_{js} = S_{ji} / \mu \qquad (11)$$

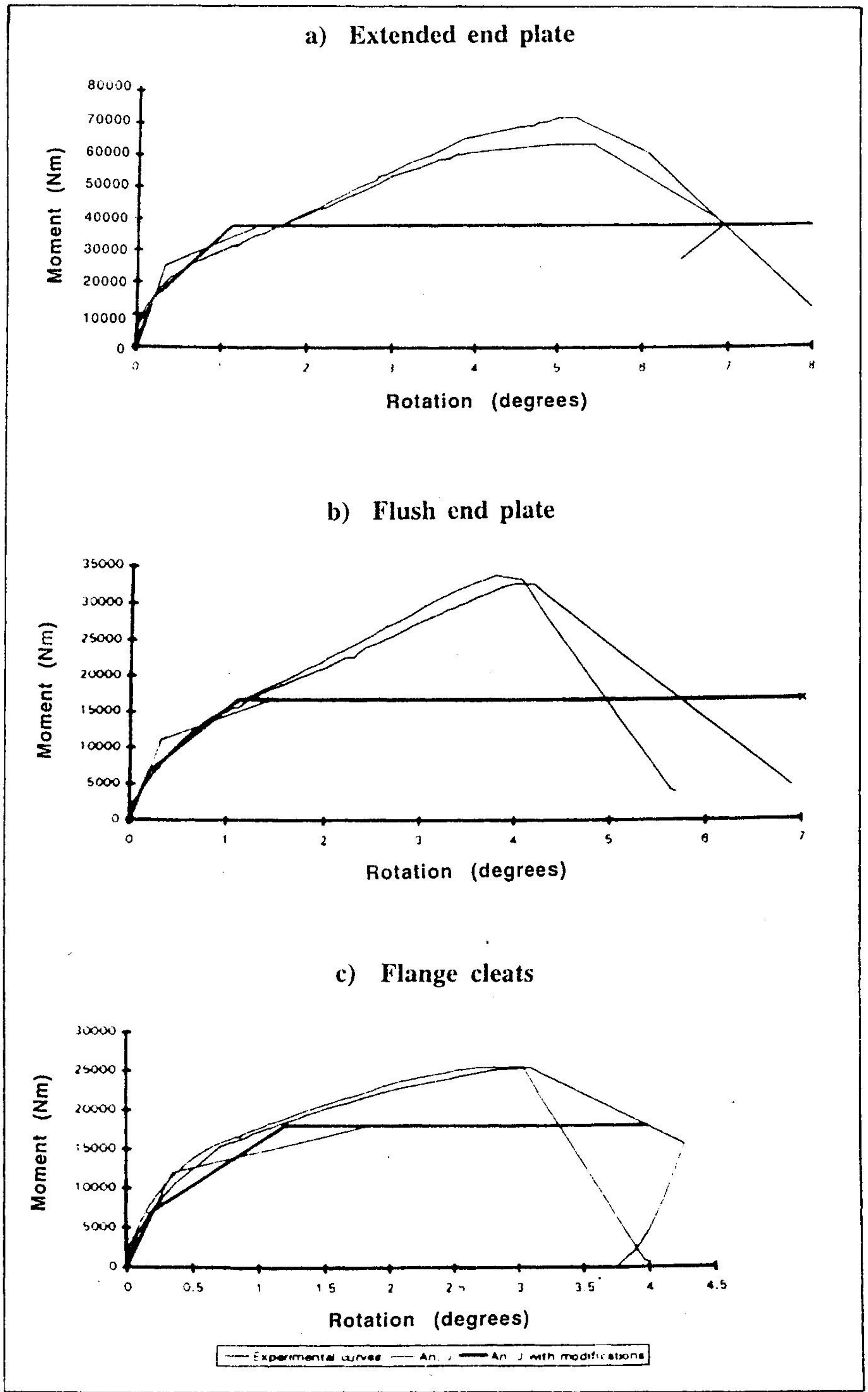

Fig. 4. Experimental curves and comparison with the curves calculated according to EC3 Annex J (with and without modifications)

Comparing the value of S_{js} (calculated according to EC3 Annex J and deriving from the value of the initial stiffness S_{ji} deduced from the calculation and numerical simulations) and the measurements on the experimental curve, leads to the results given in table 2:

	S_{ji} (in kN.m/deg)	S_{js}^{EC3} (in kN.m/deg)	S_{js}^{EXP} (in kN.m/deg)	$\mu^* = S_{ji} / S_{js}^{EXP}$
Extended end-plate	78.97	26.43	24,59	3,21
Flush end-plate	34.85	11.66	14.36	2.43
Flange cleats	37.23	10.59	16.80	2.21

Table 2 : Stiffness coefficients

As a first approximation and to keep a simple solution, it's possible to give the following ratios :

$$M_e = M_{Rd}/2,5 \qquad (12)$$
$$S_{js} = S_{ji}/\mu \qquad \text{with} \qquad \mu = 2.3 \qquad (13)$$

Figure 4 shows the tri-linear curves drawn with these values.

7. Influence of the membranar effect on frame response.

As already said, membranar effects develop in the bended face of the column section. The ultimate moment M_u is therefore largely higher than the design moment resistance M_{Rd}. However, even if this sort of connection have a large resistance reserve, the rotations which correspond to the ultimate value M_u are relatively high and not acceptable for the serviceability limit state. M_u cannot be considered for the design [7]. Nevertheless, the rotation capacity is very high.

It is important to study the influence of neglecting this so-called joint post-limit stiffness on the frame response [8]. In a plastic analyse, a yield plateau is considered, but the actual behaviour of the joint is different. That leads to a different internal forces repartition within the frame.
That also means that the load factor at the ultimate state will be higher in the reality than the one given by a plastic analyse, as well as the actual moment in the joint for a given external load.

If the structure is submitted to the ultimate loads, derived by a plastic analyse, the joint rotation is assumed to be equal to $\varnothing_l$ and the corresponding moment in the joint is equal to M_{Rd}, in case of a plastic mechanism collapse mode for the beam (see figure 5). In fact, for the same load level, because of the actual internal forces distributions, the actual joint rotation is equal to $\varnothing^*$ and the corresponding moment to M^*.

In EC 3 Annex J, a rather limited post-limit effect mainly due to strain hardening is safely considered. The ratio between the post-limit and initial stiffness considered in the code is about 1/50. When membranar effect occurs, the experimental curves shown in figure 4. give a ratio close to 1/6, which is quite different.

The influence of the post-limit effect is important for the design of the welds between the beam and the column or end-plate. EC 3 Annex J states that they have to be design for 1.4 M_{Rd} (or for the plastic design resistance of the beam flange).
If the ratio between the actual moment M^* in the joint and M_{Rd} is higher than 1.4 for braced frames, the welds design is unsafe for this particular connections and a brittle failure might occur. It is too early to give results, but studies are under developments.

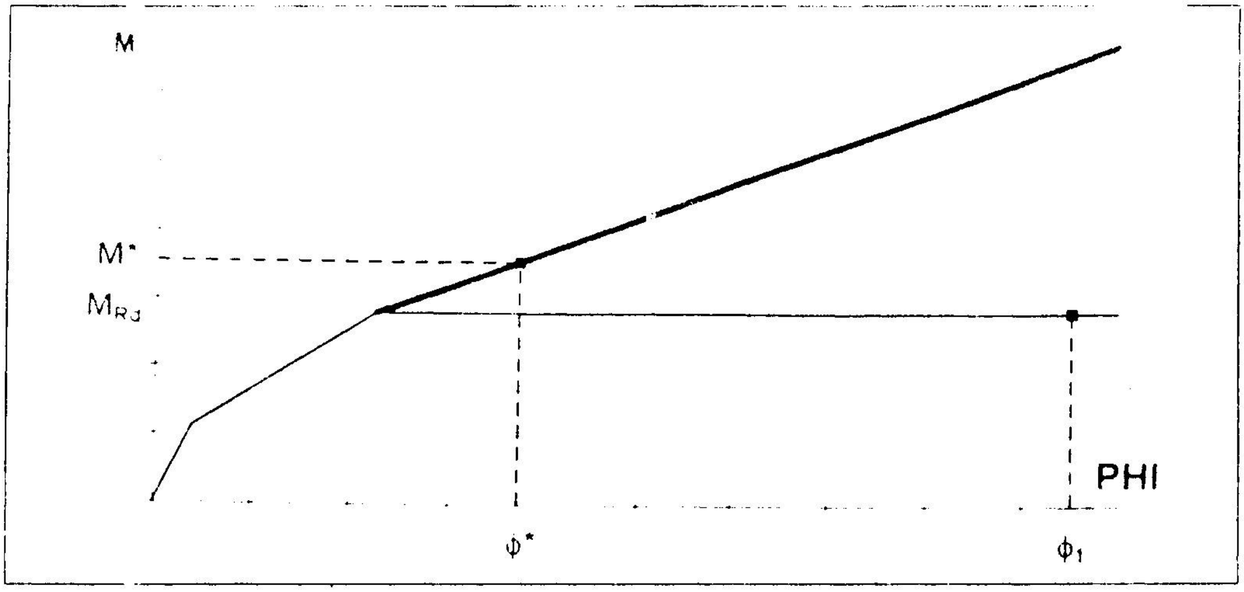

Fig. 5. Variation of the joint response with the post-limit stiffness

8. Conclusions.

The stud technique makes it easy to build a connection between a I-beam and a R.H.S. column filled with concrete. Reliable design models exist to determine a design value of the joint resistance.

The component method described in EC 3 Annex J can be extended to this sort of connections, without any difficulties, what is of great interest. However, small changes have to be made in the assembly procedure of the different components.

The only component for which the deformation cannot be derived analytically is the bended face of the column section. Numerical simulations have shown that it is possible to find the initial stiffness of the joint with a good degree of precision. The deformations of the lateral faces of the section can be neglected.

Important membranar effects occur in the face of the column. Studies are nowadays under development to ensure that it is safe to neglect that particular effect.

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